ON GUST FACTORS FOR WIND-EXCITED BUILDING RESPONSE

A Thesis Submitted

in Partial Fulfilment of the Requirements

for the Degree of

MASTER OF TECHNOLOGY

by

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to the

DEPARTMENT OF CIVIL ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY KANPUR

April 1993

CERTIFICATE

It is certified that the work contained in the thesis entitled "**On Gust Factors For Wind-Excited Building Response**" by "**K.V.S. Chandra Sekher**" has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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dedicated
to
my dear
parents and sister

ACKNOWLEDGEMENTS

I am very grateful to my teacher and advisor, Dr. V.K. Gupta for the immense help he has provided me. Working under him proved to be a very enjoyable experience.

I would like to express my thanks to all of my teachers at I.I.T., Kanpur who made the process of learning for me a very satisfying one.

I am grateful to all my classmates and friends, particularly to Pavan, Ravi, Prasad, Subbu, Gamar, Sridhar, Srikanth, Jones, Jakka, Rahul, Rajan, Rashi and Galag for being a constant source of help and encouragement. I am thankful to all of my friends at Bangalore, specially to Prasanna, Viji, Deepa, Anand and Guru for providing me with the inspiration to pursue my efforts. Finally, I am deeply indebted to my parents but for whom, I would not have been what I am today.

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ABSTRACT

A state-of-the-art review of the gust factor method for the alongwind response of structure has been presented. The provisions of this method as recommended by the Indian Standard Code, IS:875 (Part 3)–1987, have been critically examined. It is shown that the codal values may be too conservative as those are based on the assumption of wind energy spectra being independent of height above the ground. Also, the assumption of perfect correlation between the velocities on the windward and leeward faces of the building is shown to give conservative estimates. It has been shown that the use of more realistic logarithmic law for the mean wind velocity profile leads to even lower gust factors. Further, the assumptions of first mode shape being linear and that of the complete dominance of the first mode in the dynamic wind response of buildings have been justified.

CHAPTER I
INTRODUCTION

1.1 General Review

It is now a well established practice in the design of almost all-above-the-ground structures to appropriately account for the wind loading. For the structures highly exposed to the wind, such as skyscrapers, tall masts, long span bridges, radio telescopes etc., this may be a major factor governing the initial cost and safety of the structure. Traditionally, wind pressures were calculated on the assumption that the fluctuations in the wind velocity could be safely disregarded and that the velocity could be regarded as invariant of time and space. Thus, the wind loading was idealized in form of a static loading. The aerodynamic coefficients used to describe this, such as the coefficients of pressure and drag, could be determined directly from the experimental observations.

Although this procedure was simple, and depended on the data which could be easily obtained, it was far from being realistic. The development of modern materials and construction techniques has resulted in the emergence of new generation of structures that are very flexible, light in weight and are having low damping. The natural frequencies of these structures may lie in the same range as the average frequency of the powerful gusts and therefore, these structures are likely to experience large resonant motions under severe wind loads. Thus, it has become necessary to develop methods which include the dynamic behaviour of these structures due to the gusts. The past one and half decade has seen the development of several approaches to deal with this need.

Wind loading of structures is unique as it is a multidimensional, random process. This random nature of wind has been considered by several workers in the calculation of the response of structures to wind. Pioneering work in the dynamic response of structures to wind was done by Davenport (1961b). He used the statistical concepts of a stationary time series to determine the response of a structure to a turbulent gusty wind. Based on these concepts, he was able to develop a procedure for estimating the alongwind response of single-degree-of-freedom (SDOF) systems. Later, he extended these concepts to the continuous systems (Davenport (1967)). Vickery (1970) subsequently developed a procedure similar to that proposed by Davenport which allowed for more flexibility with respect to the choice of certain meteorological parameters.

Both the above procedures, however, suffered from a drawback that the fluctuating pressures on windward and leeward faces of the structure were assumed to be perfectly correlated. Vellozzi and Cohen (1968) developed a modified procedure by which this imperfect correlation was accounted for by incorporating a reduction factor. However, Simiu (1974) showed that this factor caused underestimation in the variance of the building response. He then proposed a different method to account for the alongwind correlation. But his method did not consider the effects of structural dimensions and frequency content of the loading in estimating the alongwind correlation. At a later stage, (Simiu (1980)) introduced a new expression for the alongwind correlation incorporating these effects also.

In the procedures of Davenport, Vickery and Vellozzi as mentioned above, it has been assumed that the characteristics of turbulence do not vary

with height above the ground. According to the results of modern meteorological research (Simiu (1974)), however, the energy of turbulent fluctuations that causes resonant oscillations in the tall buildings decreases significantly at higher elevations. Simiu (1974) proposed a energy spectrum for the fluctuations in wind velocity which accounted for the height dependence in calculation of the spectrum energy in any frequency band. On the basis of his formulation, simple procedures were developed from which rapid manual calculations for alongwind response could be made. Simiu (1980) developed a series of graphs to determine the alongwind response of structure. Using the same formulation as in Simiu (1980), Solari (1982) developed a closed form solution for the alongwind response by approximating the results of the numerical integrations by simple functions. All the procedures developed for the calculation of gust factors as above assumed that the modes other than the fundamental mode contributed negligibly to the dynamic response. Vickery (1970) and Simiu (1974) found this to be justified in case of the second mode of buildings.

Response spectrum techniques which have been popular in Earthquake Engineering, have also been used to determine the alongwind response of buildings. Torkamani (1985) developed response spectra assuming the gust to be sinusoidally fluctuating and perfectly coherent in space. Solari (1988) developed a more elegant procedure which took into account the correlation and random nature of wind. In this model, wind has been idealised as consisting of a mean wind profile, upon which an "equivalent" turbulent configuration which is perfectly coherent in space is superimposed. The criteria of equivalence has been formulated by defining a fictitious velocity fluctuation process, which is a random function of time only, such that the corresponding power spectrum of modal

force optimally approximates the corresponding modal spectrum obtained from the actual turbulence configuration. Using this model of wind, Solari (1989) developed a wind response spectrum .

All of the above mentioned procedures involved calculations in the frequency domain. Time domain analysis of the alongwind response has also been carried out by some research workers (e.g., Vaicatis et al. (1975), Jayachandran et al. (1976)). Their results seem to agree with those obtained from the frequency domain analysis. However, time domain analysis is computationally intensive and difficult to be adopted in practice.

Tall buildings are bluff, and hence, cause the flow to undergo separation rather than following the body contour. The wake flow thus created behind a building is asymmetrical. It is primarily due to this asymmetry that buildings exhibit an acrosswind response, in addition to the contributions of lateral turbulent fluctuations in the incoming flow. Due to the complexity of wind-structure interaction, expressions based on first principles, for estimating the acrosswind response of tall buildings, do not currently exist. Some experimental work has however been done in this direction. Kareem (1982), Tallin and Ellingwood (1985) determined the acrosswind response of structures by using force spectra measured from the wind tunnel tests. Their results have showed that for tall buildings (height/breadth ratio > 4), the acrosswind response is very important and it might also dominate the alongwind response. Kareem (1981) calculated the contributions of various modes in the acrosswind response. His results showed that for the acrosswind response, as in the case of alongwind response, the contributions to the total building response from the modes higher

than the fundamental mode are negligible.

Structures under the effect of wind also experience torsion, since the center of mass and/or the elastic center do not coincide with the instantaneous point of application of the aerodynamic loads. The first attempt at studying the torsion induced in buildings by fluctuating wind loads in an analytical manner was reported by Patrickson and Friedman (1979). Later Safak and Foutch (1981) also presented potentially useful methods for estimating the torsional response. From their results, it was clear that even in symmetrical structures, torsional forces do exist due to the non-homogeneity of the wind pressures across the building surface. Kareem (1985), Tallin et al. (1985) determined torsional force spectra for symmetric buildings from the wind tunnel tests. Their results confirmed the earlier observations that the torsional response contributes significantly towards the overall dynamic response of symmetric buildings. Including eccentricity between mass and/or stiffness and resultant aerodynamic force further compounds this effect. The importance of torsional response was further emphasized by the fact that torsional motion was evident at a much lower level than the lateral transational motion. Nevertheless, the provisions for design torsional loads are yet to be incorporated in the present codes of practice. This has not been done so far due to the complexity of wind-structure interaction, variability in the geometric shapes, and due to the effects of dynamic characteristics and configuration of the surrounding buildings.

Attempts have also been made to develop a unified method to estimate the alongwind, acrosswind and torsional responses of buildings. Solari (1985) developed a mathematical model to predict the overall response. But the method

suffered from the drawback that it was applicable only to the square shaped buildings with wind blowing normal to the building. Torkamani and Pramano (1985), Tallin et al. (1985) have determined the overall response from force spectra determined from the wind tunnel tests. Their results have shown that the coupling between lateral and torsional modes can be significant if the modes are very close. Foutch (1981) has shown that even if the frequencies of these modes are not close, the coupling effects can be significant when the torsional frequency is near the spectral peak (i.e. around 0.1 Hz). Islam et al. (1992) have shown that for non-zero eccentricity in the alongwind direction, the lateral and torsional responses tend to be correlated. Interestingly, their results have showed that even for the uncorrelated torsional and lateral forces, the combined response can be much higher than that for the correlated forces. This is due to the increase in contribution of acrosswind torque covariance term at large negative eccentricities.

Solari (1993a) introduced a new method for the alongwind response where the finite duration of record T and the finite response time of the wind velocity measuring instrument τ, are taken into account while calculating the expected values. The effect of T is to cut off the spectra at certain lower frequency while τ cuts off the spectra at some higher frequency. Using these parameters, Solari (1993b) modified his earlier response technique (Solari (1988)) by allowing the user to define his own mode shape, velocity profile etc. Thus the new method is more flexible than his previous method but is as computationally intensive as his earlier one.

The existing codes of practice on wind response of buildings consider

only the alongwind response of structures. The Indian Standard code, (I.S. 875–1987) recommends the gust factor method for calculation of the dynamic wind loads on buildings. This method calculates the peak response of a structure to turbulent, gusty wind, and is essentially based on the works of Davenport (1961b) and Vickery (1970). Based on the results of recent research, several workers have proposed a modified procedure to calculate the gust factors (e.g. Simiu (1980), Solari (1986)). In this study, attempts are made to calculate the gust factors for tall buildings incorporating these developments, and to compare those with the gust factors specified by the code. For this, complete theoretical formulation of the gust factors is also presented. The code provisions have been examined critically in the light of assumptions on which those gust factors are based.

1.2 Organization

This work has been presented in three chapters following this chapter.

In Chapter II, formulation of the gust factor approach for estimating the largest peak response of a building to gusty wind has been presented.

In Chapter III, the formulation developed in Chapter II has been used and gust factors have been obtained by using different expressions, (i) as per the code, and (ii) as per the state-of-the-art, for the alongwind correlation, velocity spectra and wind profile. It has been shown that the gust factors obtained by using the code are too conservative.

A brief summary and conclusions of this study are presented in Chapter IV.

CHAPTER II

FORMULATION OF THE GUST FACTOR METHOD

2.1 Background

The response of a structure to a fluctuating wind has been calculated by various methods like gust factor method (Davenport (1961b), Simiu (1973)), equivalent wind spectrum technique (Solari (1988)) etc. Among them, the gust factor method is the most widely adopted method due to its simplicity. The basic theory of this method has been presented by Davenport (1961b). The salient features of this method are outlined here.

2.2 Structure of Gusty Wind

General expression for the force induced on a structure due to the unsteady flow is

$$F(t) = A\left[\frac{1}{2}\rho c_D u^2(t) + c_m \rho \frac{A_o}{D}\frac{du(t)}{dt}\right] \tag{2.2.1}$$

where $F(t)$ is the force at any instant t, due to velocity $u(t)$; A is the area over which the force $F(t)$ acts; ρ is the density of the fluid; c_m and c_D are the coefficients of virtual mass and drag respectively; D is the diameter or any such typical dimension (such as breadth); and, A_o is the reference area for the virtual mass (generally $\pi D^2/4$).

The coefficient of mass c_m is intended to account for the effects linked to fluid acceleration. It is significant in those cases where the fluid mass is appreciable relative to the body mass, particularly if the fluid is relatively dense, e.g. in case of water. In wind engineering, however, the entire term containing

therefore not retained in the following formulation.

Consider a SDOF system for which the equation of motion is described by

$$m\ddot{y} + c\dot{y} + ky = F(t) \tag{2.2.2}$$

where m is the mass of the system, c is the damping coefficient, and k is the stiffness. $F(t)$ is the dynamic load as described by Eq. (2.2.1), and y is the displacement response to this load.

For any fluctuating input velocity $u(t)$, the solution of Eq. (2.2.2) gives the displacement, $y(t)$ of the system. If the input velocity is a random process, it becomes necessary to determine the statistical properties of the wind velocity to compute the system response. It is known that the wind velocities can be taken to be distributed according to the Gaussian law. Let the velocity $u(t)$ at any instant t be written as

$$u(t) = \bar{u} + u_t \tag{2.2.3}$$

where, $\bar{u}$ is the mean velocity and u_t is the fluctuating wind velocity with zero mean. Then,

$$\bar{u} = \frac{1}{T}\int_0^T u(t)\,dt, \tag{2.2.4}$$

T being the duration of the record over which the velocity was measured. If $S_u(n)$ represents the energy spectrum and, σ_u, the standard deviation of u_t, then,

$$\sigma_u = \left[\int_0^\infty S_u(n)\,dn\right]^{1/2}. \tag{2.2.5}$$

Once the mean value and the standard deviation of velocity are known, its probability distribution is completely defined.

2.3 Mean and Fluctuating Response

Using an analogous expression as for velocity, the fluctuating wind load $F(t)$ can be expressed as

$$F(t) = \bar{F} + f_t \tag{2.3.1}$$

where $\bar{F}$ is the mean wind load and f_t is the fluctating wind load with the zero mean. The mean value, $\bar{F}$ ($\bar{F} = \frac{1}{T}\int_0^T F(t)\,dt$) of $F(t)$ can be written as

$$\bar{F} = \frac{1}{2}\rho c_D (\bar{u})^2 \left[1 + \frac{\overline{u_t^2}}{(\bar{u})^2}\right], \tag{2.3.2}$$

and the fluctuating part as

$$f_t = \frac{1}{2}\rho c_D (\bar{u})^2 \left[2\frac{u_t}{\bar{u}} + \frac{u_t^2 - \overline{u_t^2}}{(\bar{u})^2}\right]. \tag{2.3.3}$$

In the high winds, which are usually of great interest in wind engineering, $(u_t/\bar{u})^2$ and $\overline{u_t^2}/(\bar{u})^2$ are very small. They are therefore neglected, as the error involved in neglecting them is less than 2 percent (Scanlan (1986)). Hence, the above expressions for the mean and fluctuating components of force reduce to

$$\bar{F} = \frac{1}{2}\rho c_D \bar{u}^2 \tag{2.3.4}$$

and

$$f_t = \rho c_D \bar{u} u_t. \tag{2.3.5}$$

Thus, the relation between the fluctuating force and velocity becomes linear. In addition to this, for a linear structure, the relation between deflection and force is also linear. Consequently, the probability distributions of velocity, force and deflection may be assumed to be similar, namely Gaussian.

In a fluctuating flow, the coefficient of drag, c_D is not constant and is a function of the frequency, n. Hence, its effect is to be considered in the calculation of force spectra, $S_f(n)$ of f_t as

$$S_f(n) = [\rho \bar{u} c_D(n)]^2 S_u(n). \tag{2.3.6}$$

Accordingly, the standard deviation, σ_f of f_t can be written as

$$\sigma_f = \rho \bar{u} \left[\int_0^\infty c_D^2(n) S_u(n)\, dn \right]^{1/2}$$

or on using Eq. (2.3.4),

$$\sigma_f = \bar{F} \left[\int_0^\infty \chi^2(n) \frac{S_u(n)}{\bar{u}^2}\, dn \right]^{1/2} \tag{2.3.7}$$

where $\chi(n) = 2c_D(n)/c_D(0)$, $c_D(0)$ being the drag coefficient corresponding to the mean flow. In a similar manner, the mean response, $\bar{y}$ and its variance, σ_y can be computed from

$$\bar{y} = \frac{\bar{F}}{k} \tag{2.3.8}$$

and

$$\sigma_y = \bar{y} \left[\int_0^\infty \chi^2(n) n_o^2 |H(n)|^2 \frac{S_u(n)}{\bar{u}^2}\, dn \right]^{1/2} \tag{2.3.9}$$

where $H(n)$ is the complex frequency response function defined by

$$H(n) = \frac{1}{4\pi^2(n_o^2 - n^2 + 2i\zeta n_o n)}. \tag{2.3.10}$$

Here, n_o represents the natural frequency of the oscillator. Eqs. (2.3.8) and (2.3.9) completely specify the statistical distribution of oscillator response in terms of the mean wind velocity and the velocity spectrum, from which its largest peak response can be determined.

2.4 Determination of Largest Peak Response

The probability density function for the distribution of peaks for a zero mean, stationary, Gaussian process, $y(t)$ with standard deviation, σ_y is given by (Davenport (1961b)) as

$$q_y(\eta) = \frac{\nu T}{\sigma_y} \eta e^{-\eta^2/2\sigma_y^2} \tag{2.4.1}$$

where, ν is the average frequency of positive crossings i.e.

$$\nu = \left[\frac{\int_0^\infty n^2 S_y(n)\, dn}{\sigma_y^2} \right]^{1/2}, \tag{2.4.2}$$

and T is the duration of wind loading. It may be noted that for the narrow band processes, νT approaches the total number of peaks from above. The mean, $\bar{\eta}$ and standard deviation, σ_η of this distribution for large values of νT are found to be

$$\bar{\eta} = \sigma_y g(\nu T) \tag{2.4.3}$$

with $g(\nu T)$ being the peak factor given by

$$g(\nu T) = \left[\sqrt{2 \ln \nu T} + \frac{0.5772}{\sqrt{2 \ln \nu T}} \right] \tag{2.4.4}$$

and

$$\sigma_\eta = \frac{\pi}{\sqrt{6}} \frac{1}{\sqrt{2 \ln \nu T}}. \tag{2.4.5}$$

The form of this distribution and its relation to the statistics of the parent population is as shown in Fig 2.1. As can be seen from the figure, for large values of νT (i.e. for large number of peaks), the distribution of peak values is a narrow one. In practical cases, 95 percent of the peak values lie within an interval much less than half the standard deviation, σ_y of the parent population on either side of the mean peak value, $\bar{\eta}$ (Davenport (1961b)). Hence, not much error is

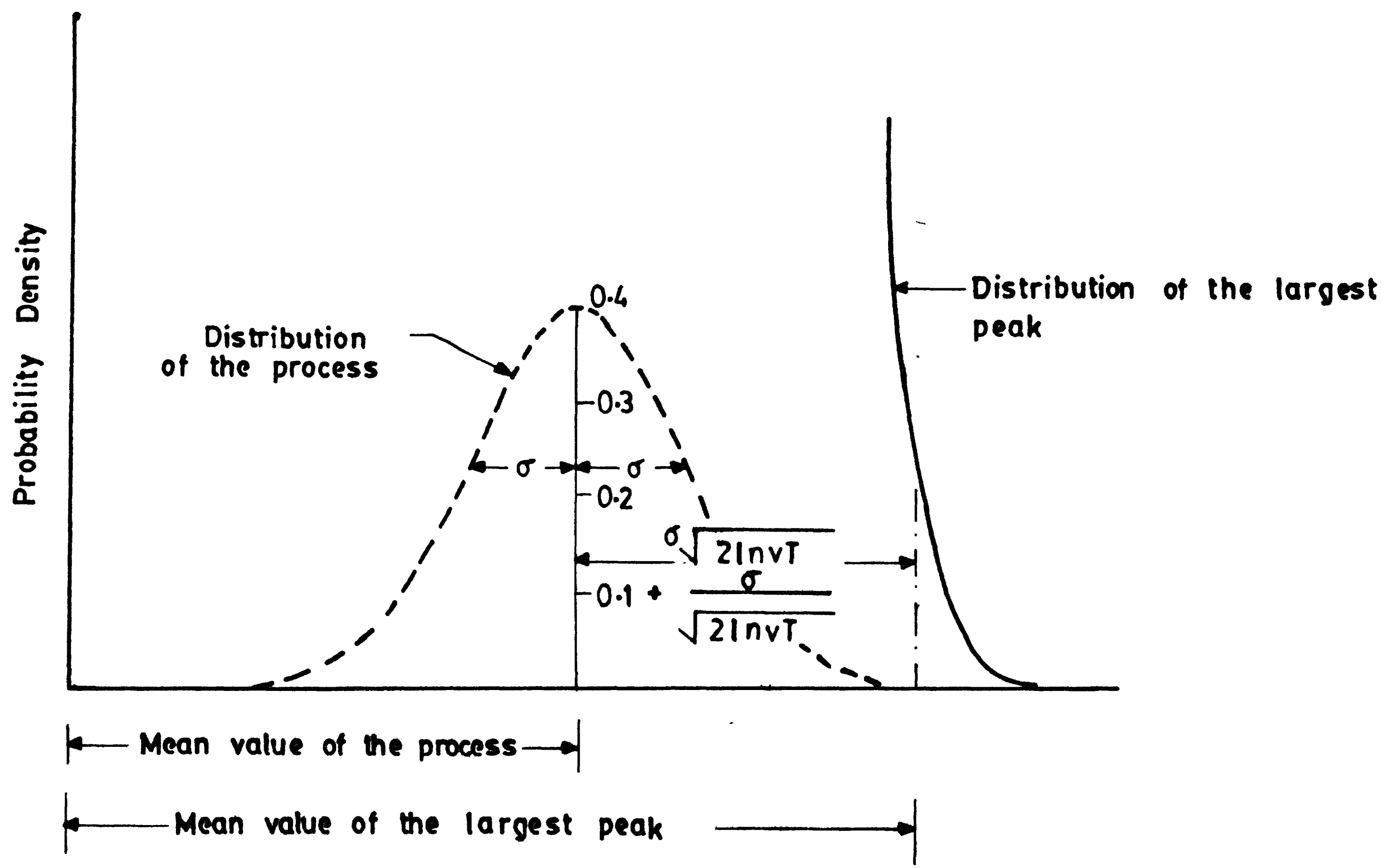

Fig. 2·1 Distribution of Maximum Peak Values for a Gaussian Process

involved if this small deviation of peak values is disregarded and the peak value is taken equal to the mean peak value irrespective of the desired probability of exceedance. In that case, the largest peak amplitude can be written as

$$\begin{aligned} y_p &= [\bar{y} + g(\nu T)\sigma_y] \\ &= \bar{y}\left[1 + g(\nu T)\frac{\sigma_y}{\bar{y}}\right] \end{aligned} \tag{2.4.6}$$

The quantity, $[1 + g(\nu T)\sigma_y/\bar{y}]$ is called the gust factor and it measures the amplification of the mean system response.

2.5 Gust Factors for Continuous Systems

The basic theory for the calculation of gust factors for continuous systems remains the same as for the SDOF systems. In addition, the following assumptions are made :

(a) The load $q(x, z)$ per unit area, projected on a plane normal to the mean wind can be expressed in the form

$$q(x, z) = c_p(x, z)\rho\frac{u^2(x, z)}{2} \tag{2.5.1}$$

where $c_p(x, z)$ and $u(x, z)$ are respectively the pressure coefficient and the instantaneous velocity at co-ordinates (x, z) on the plane normal to the flow.

(b) The response of the structure is primarily in the fundamental mode for both mean and fluctuating components.

(c) The mean velocity component, $\bar{u}(x, z)$ is a function of coordinate z only.

Consider a building rectangular in plan, as represented in Figure 2.2. Let the wind pressures on the windward and leeward sides of this building be

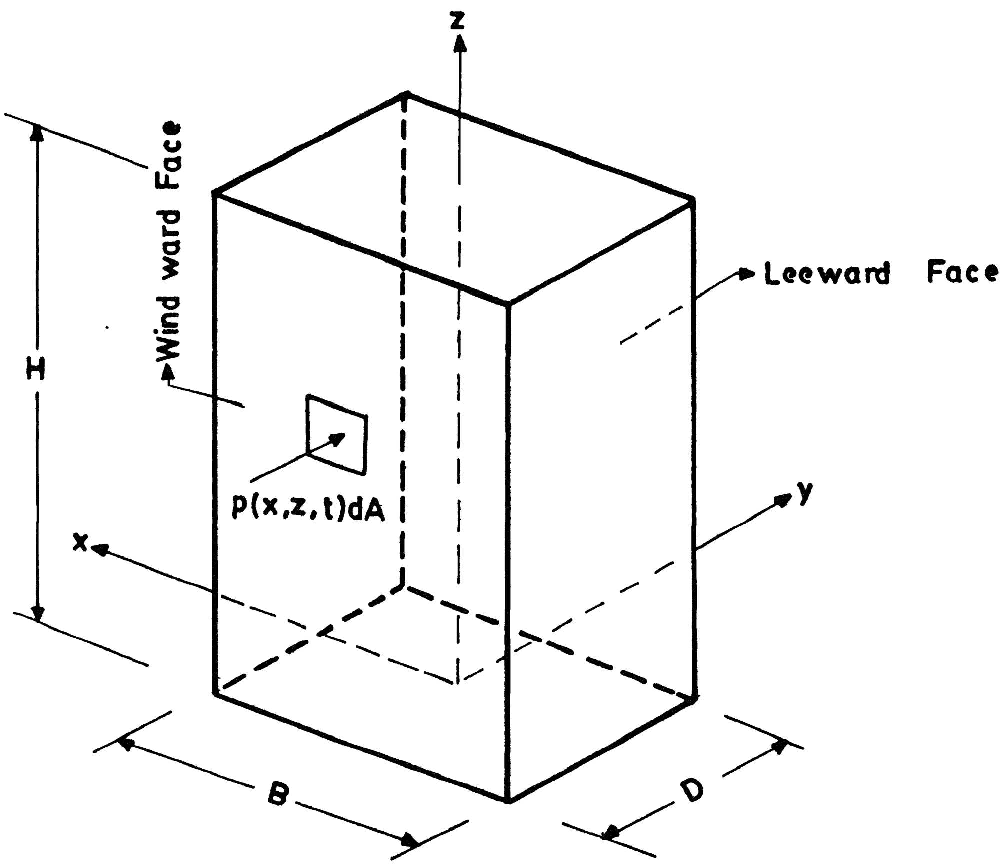

Fig. 2.2 Schematic View of the Building

denoted by $p_w(x,z)$ and $p_l(x,z)$ respectively. As in the case of SDOF systems, the pressures, p_w and p_l can also be expressed as sums of mean and fluctuating components, i.e.

$$p_w(x,z) = \bar{p}_w(x,z) + p'_w(x,z), \tag{2.5.2}$$

and

$$p_l(x,z) = \bar{p}_l(x,z) + p'_l(x,z) \tag{2.5.3}$$

with

$$\bar{p}_w(x,z) = \frac{1}{2}c_w\rho\bar{u}^2(z) \tag{2.5.4}$$

$$\bar{p}_l(x,z) = \frac{1}{2}c_l\rho\bar{u}^2(z) \tag{2.5.5}$$

$$p'_w(x,z) = \rho c_w\bar{u}(z)u_t(x,z) \tag{2.5.6}$$

and

$$p'_l(x,z) = \rho c_l\bar{u}(z)u_t(x,z) \tag{2.5.7}$$

where, $\bar{u}(z)$ is the mean wind velocity and $u_t(x,z)$ is the fluctuating component of wind velocity, and c_w and c_l are the pressure coefficients on the windward and leeward side respectively. These coefficients are assumed to be independent of the coordinates x and z. If $\mu(z)$ represents the fundamental mode shape of the structure, then the mean wind load $\bar{Q}$ can be expressed as (Simiu (1973))

$$\bar{Q} = \int_{A_w} \bar{p}_w(x,z)\mu(z)\,dA + \int_{A_l} \bar{p}_l(x,z)\mu(z)\,dA \tag{2.5.8}$$

and, the mean response of the structure is

$$\bar{y}(z) = \frac{\bar{Q}}{(2\pi n_o)^2 M}\mu(z) \tag{2.5.9}$$

where n_o is the fundamental frequency of the structure and M is the modal mass in the fundamental mode. As explained in Appendix, the fluctuating part of

the response i.e. $y'(z)$ for a linear elastic structure is described by the spectral density function, $S_y(z,n)$ as

$$S_y(z,n) = \frac{\mu^2(z)|H(n)|^2}{M^2} \int_A \int_A \mu(z_1)\mu(z_2)S_p(M_1,M_2;n)\,dA_1\,dA_2, \qquad (2.5.10)$$

where, A is the total external area of the structure and $S_p(M_1,M_2;n)$ is the cross-spectral density of the pressure fluctuations acting at points M_1 and M_2 of elemental areas dA_1 and dA_2, and of ordinates z_1 and z_2 respectively. The function $S_p(M_1,M_2;n)$ is, in general, a complex function. It has been found from the experiments (Davenport (1967)), however, that the imaginary part of this function is negligible. The real part is given by the expression (see Appendix)

$$S_p(M_1,M_2;n) = \rho^2 c\bar{u}(z_1)\bar{u}(z_2)S_u^{1/2}(z_1,n)S_u^{1/2}(z_2,n)R(x_1,x_2,z_1,z_2;n) \quad (2.5.11)$$

where

$$\begin{aligned} c &= c_w^2 \quad \text{if points } M_1 \text{ and } M_2 \text{ are on the windward side} \\ &= c_l^2 \quad \text{if points } M_1 \text{ and } M_2 \text{ are on the leeward side} \qquad (2.5.12) \\ &= N(n)c_w c_l \quad \text{if points } M_1 \text{ and } M_2 \text{ are on the opposite sides.} \end{aligned}$$

$N(n)$ is called the alongwind correlation factor. Here, c is not defined on the side faces as the skin friction drag is assumed to be negligible. The function $R(x_1,x_2,z_1,z_2;n)$ is the square root of the coherence function of velocities at points M_1 and M_2. Its value, as proposed by Davenport (1967), is

$$R(x_1,x_2,z_1,z_2;n) = \exp\left[\frac{-2nc_z\sqrt{(z_1-z_2)^2+(c_x b/c_z h)^2(x_1-x_2)^2}}{\bar{u}(z_1)+\bar{u}(z_2)}\right] \qquad (2.5.13)$$

where c_x and c_z are the coefficients representing the decay of velocity correlations between two points on the surface of the building in the horizontal and vertical

directions respectively. Further, $S_u(z,n)$ is the spectrum of longitudinal velocity fluctuations at z ordinate.

As the skin friction drag has been neglected, the integration in Eq. (2.5.10) is carried out only on windward and leeward faces, with areas A_w and A_l respectively, thus leading to

$$\begin{aligned} S_y(z,n) =& \frac{\mu^2(z)|H(n)|^2}{M^2}\left[\int_{A_w}\int_{A_w} \mu(z_1)\mu(z_2)S_p(M_1^w, M_2^w; n)\,dA_1\,dA_2\right. \\ &+ 2\int_{A_w}\int_{A_l} \mu(z_1)\mu(z_2)S_p(M_1^w, M_2^l; n)\,dA_1\,dA_2 \\ &\left.+ \int_{A_l}\int_{A_l} \mu(z_1)\mu(z_2)S_p(M_1^l, M_2^l; n)\,dA_1\,dA_2\right]. \end{aligned} \tag{2.5.14}$$

Denoting the first, second and third integrals by $S_{ww}(n)$, $S_{lw}(n)$ and $S_{ll}(n)$, the above equation can be rewritten as

$$S_y(z,n) = \frac{\mu^2(z)|H(n)|^2}{M^2}\left[S_{ww}(n) + 2S_{lw}(n) + S_{ll}(n)\right]. \tag{2.5.15}$$

In the above derivation, it has been implicitly assumed that the velocity distribution at any time on the leeward side is same as that on the windward side. Results of the full scale measurements suggest, however, that the velocity fluctuations on leeward side are smaller than that on the windward side (Scanlan (1986)). Thus the use of Eq. (2.5.11) is conservative from the structural safety viewpoint. Now, since $\sigma_y(z) = \int_0^\infty [S_y(z,n)\,dn]^{1/2}$, Eqs. (2.5.9) and (2.5.15) can be used in Eq. (2.4.6) to obtain

$$G(z) = 1 + g(\nu T)(2\pi n_o)^2 \frac{\left\{\int_0^\infty |H(n)|^2\left[S_{ww}(n) + 2S_{lw}(n) + S_{ll}(n)\right]\,dn\right\}^{1/2}}{\int_{A_w}\bar{p}_w(x,z)\mu(z)\,dA + \int_{A_l}\bar{p}_l(x,z)\mu(z)\,dA}. \tag{2.5.16}$$

wind profile, and by using Eq. (2.5.16), the value of the gust factor $G(z)$ can be determined.

CHAPTER III

REVIEW OF CODE PROVISIONS FOR GUST FACTORS

3.1 Background

The Indian Standard code on wind loading (I.S. 875–1987) prescribes the gust factor approach for the estimation of wind loads on the flexible structures. Certain assumptions are made in the calculation of gust factor as recommended by the code. Here, the validity of these assumptions has been critically examined in the light of recent developments. For this, gust factors for some example buildings have been calculated as per the code, and compared with the values obtained by using improved expressions for longitudinal wind spectra and alongwind correlation. In addition to this, certain fundamental assumptions used in the formulation of gust factor approach (see section 2.5) have been validated.

The expression for gust factor, G is conventionally taken as in Eq. (2.4.6). It has been shown by Vickery (1970), Simiu (1980) that the value of peak factor $g(\nu T)$ varies in a narrow range of 3.4–3.7. It is the ratio of the standard deviation of the fluctuating part of the response to the mean response i.e., $\sigma_y(z)/\bar{y}(z)$, hereafter called as the gust ratio, which mainly causes variations in the gust factors (Simiu (1974), Lee (1990)), depending on the different velocity spectra and the alongwind correlation factors used by various authors. For a constant value of peak factor (Vickery (1970)), it can be seen that the variation in gust factors will be truly reflected by the variation in gust ratios. The variation in gust factors have thus been studied here by considering the variation in gust ratios.

3.2 Assumptions in the I.S. Code

The assumptions made in the calculations of gust factors in section 7 (dynamic effects) of I.S. 875–1987 are

1. The value of alongwind correlation coefficient $N(n)$ (see Eq. (2.5.12)) is equal to unity.

2. The spectrum of velocity fluctuations, $S_u(z, n)$ (as used in Eq (2.5.11)) is independent of height parameter, z.

3. The variation of mean wind speed with height follows the power law.

Each of these assumptions are critically examined in the following section with the help of four example buildings. The dimensions of these buildings are as follows :

Example Building #	Breadth in m	Height in m	Depth in m
1	30	50	30
2	30	120	30
3	30	150	30
4	60	365	60

Example buildings # 1 and 4 are the same as considered by Vickery (1970). These buildings are assumed to be located in a densely populated urban area, since this type of terrain is associated with more pronounced fluctuations in the wind velocity due to the increased surface roughness. This leads to greater contribution of the dynamic component in the structural response. Further, the basic wind speed, as defined in the code, has been assumed to be 50 m/sec.

For the calculation of gust ratios using the height-dependent velocity spectra, the value of friction velocity u_* is needed. Several authors (e.g., Simiu (1974), Solari (1982)), have specified the values of u_* for various terrain categories. For urban areas, this value of u_* has been specified to be around 2.26 m/sec. Hence, this value of friction velocity has been adopted in this study.

3.2.1 Alongwind Correlation

The I.S. code assumes that the wind pressures on windward and leeward faces of the structure are perfectly correlated. In other words, the value of $N(n)$ in Eq. (2.5.12) is assumed to be unity. Experiments and full scale tests have indicated that the value of $N(n)$ is less than unity (Simiu (1974)). For this study, the expression proposed by Vellozi (1968) for $N(n)$ is adopted where,

$$N(n) = \frac{1}{\xi} - \frac{1}{2\xi^2}(1 - e^{-2\xi}) \tag{3.2.1.1}$$

with

$$\xi = \frac{15.4 n \Delta x}{\bar{U}} \quad . \tag{3.2.1.2}$$

Here, $\bar{U}$ is the mean wind velocity at $z = 2H/3$, and Δx is the smallest of the dimensions B, H and D, B being the breadth, H the height, and D the depth of the structure. It has been found that this expression matches with the experimental results fairly well (Scanlan (1986)).

For the purposes of comparison here, the gust ratios for example buildings # 3 and 4 have been computed using the formulation of Chapter II (see section 2.5) . The natural frequencies of the building have been varied from 0.05 to 0.5 Hz, and the damping ratio ζ has been taken as 0.01. Two cases have been considered, i) for correlation as per Eq. (3.2.1.1) and ii) for perfect correlation.

Table 3.1 and Figure 3.1 give the results of gust ratios for these two cases. It is seen that the imperfect correlation (as in Eq.(3.2.1.1)) may cause a variation in gust ratios from as low as 4 percent to as high as 25 percent, the values for perfect correlations being on the conservative side. However, irrespective of the magnitude of this variation, it is quite unrealistic to assume the alongwind correlation as unity as it is against the basic theory of fluid flow (Simiu (1974)).

For simplicity in calculations, Simiu (1973) has suggested the alongwind correlation factor to be approximated by a constant value of 0.2. However, it has been seen in the case of example buildings that $N(n)$ may vary from 0.5 to 1.0. Due to this, the more complicated description of alongwind correlation as in Eq. (3.2.1.1) will be retained for the further calculations of gust ratios.

3.2.2 Velocity Spectra

The spectra, $S_u(z,n)$ of velocity fluctuations as considered in the code is given by (Vickery (1970))

$$S_u(z,n) = 4.0\kappa\bar{u}_1^2\frac{x}{(1+x^2)^{4/3}} \tag{3.2.2.1}$$

where

$$x = \frac{nL(H)}{\bar{u}(H)}, \tag{3.2.2.2}$$

and κ is a factor which depends on the surface roughness. $\bar{u}_1$ is the mean velocity at 10 m height; $L(.)$ is a turbulence scale which depends on the terrain and height H of the structure; and $\bar{u}(H)$ is the mean wind velocity at the top of the structure. As per the above expression, the velocity fluctuations are independent of parameter, z. However, as shown by (Davenport (1961a)) (see Fig. 3.2), different spectra will be obtained for various values of z. Further, the spectra

Table 3.1 Effect of Alongwind Correlation on Gust ratios ($\zeta = 0.01$)

Freq. in Hz	Building # 3 Code	Building # 3 Eq. 3.2.1.1	% Error	Building # 4 Code	Building # 4 Eq. 3.2.2.1	% Error
0.05	1.470	1.316	11.7	0.846	0.815	4.0
0.10	1.015	0.953	11.2	0.519	0.496	4.63
0.15	0.786	0.709	10.86	0.398	0.342	14.37
0.20	0.641	0.564	13.65	0.336	0.281	19.57
0.25	0.556	0.474	17.30	0.296	0.244	21.31
0.30	0.500	0.414	20.77	0.276	0.220	25.45
0.35	0.476	0.369	29.00	0.264	0.209	26.32
0.40	0.439	0.333	31.83	0.250	0.208	20.19
0.45	0.406	0.312	30.13	0.245	0.205	19.51
0.50	0.384	0.297	29.30	0.240	0.204	17.68

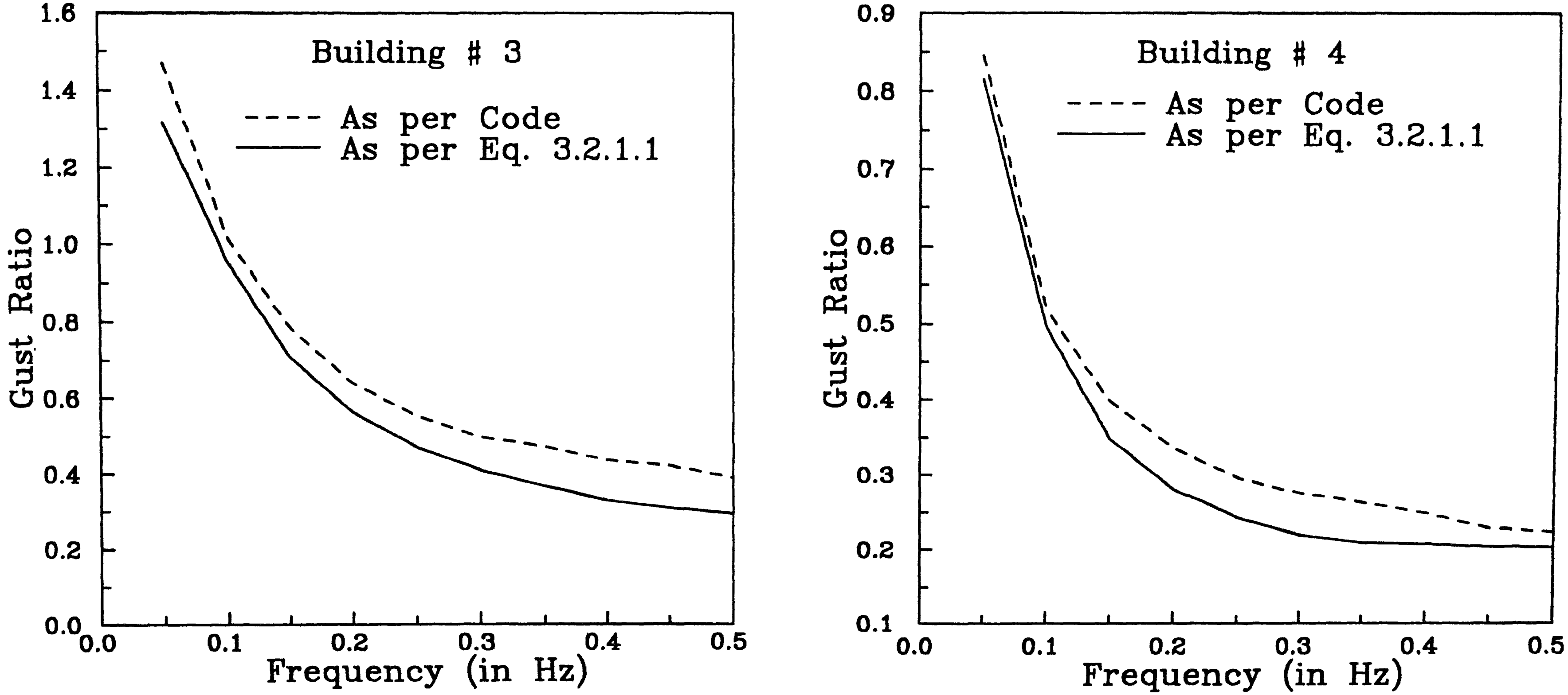

Fig. 3.1 Effect of Alongwind Correlation on Gust Ratio ($\zeta = 0.01$).

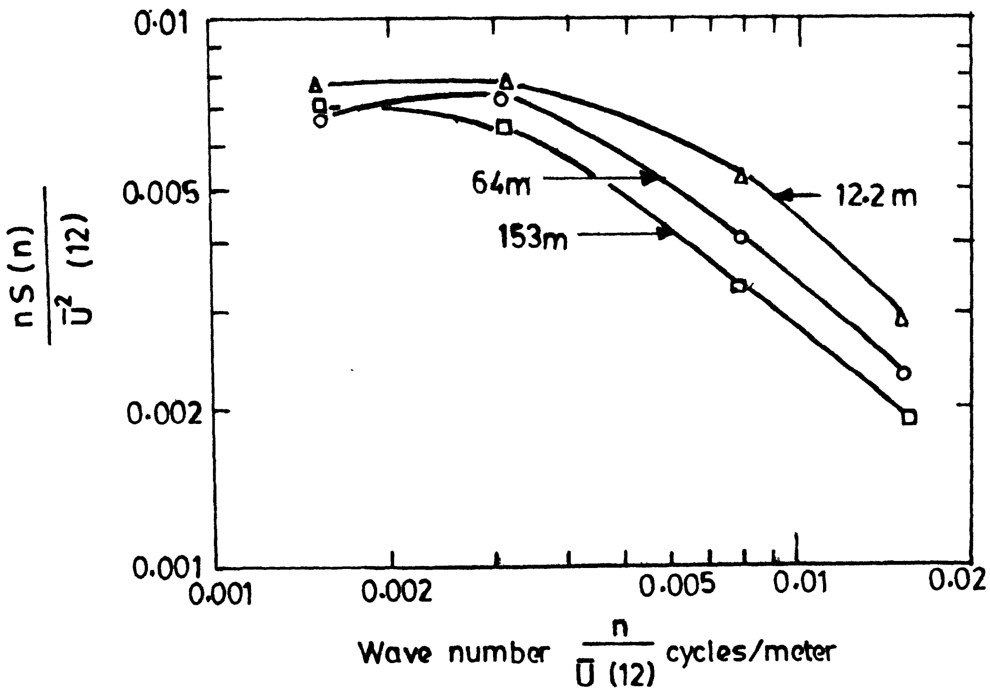

Fig. 3·2 Variation of Velocity Spectrum with Height

at lower heights are likely to have greater amplitudes for a given frequency, n. Davenport (1961a) ignored the observed variations in spectral amplitudes with height, assuming that these variations might be of the same order as that of the errors in the estimation of the spectra. Thus, the above expression for spectra was obtained by averaging the spectra at various heights above the ground. Simiu (1974) has shown that this assumption of Davenport is in error, and has proposed the following height-dependent spectra:

$$S_u(z,n) = \frac{200f}{(1+50f)^{5/3}} \frac{u_*^2}{n} \tag{3.2.2.3}$$

where

$$f = \frac{nz}{\bar{u}(z)}, \tag{3.2.2.4}$$

u_* is the friction velocity and $\bar{u}(z)$ is the mean velocity at height z above the ground. This characterisation of spectra has, on an average, been shown to agree very well with the experimental data (Simiu (1974)) in the high frequency range with $f > 0.2$. In the low frequency range, however, the velocity spectra cannot be described by a single expression as above. Since, most of the buildings have their natural frequencies in the range $0.1 < n < 1.0$, the spectra as in Eq. (3.2.2.3) can be used for the entire frequency range, including the low frequencies, without causing appreciable errors.

The gust ratios here have been calculated for all the four example buildings with varying natural frequencies in two cases : i) the spectra is height-independent as in Eq. (3.2.2.1) and, ii) the spectra is height-dependent as in Eq. (3.2.2.3). Two damping ratios, $\zeta = 0.01$ for steel structures and $\zeta = 0.016$ for concrete structures have been considered. The results have been presented in Tables 3.2–3.5. These results have also been shown in the form of curves showing

Table 3.2 Effect of Different Velocity Spectra on Gust Ratios
(Example Building #1)

Freq. in Hz	$\zeta = 0.010$ Eq. 3.2.2.1	$\zeta = 0.010$ Eq. 3.2.2.3	% Error	$\zeta = 0.016$ Eq. 3.2.2.1	$\zeta = 0.016$ Eq. 3.2.2.3	% Error
0.05	3.350	1.570	113.37	2.688	1.272	111.32
0.10	2.590	1.088	138.05	2.093	0.903	131.78
0.15	2.030	0.861	135.77	1.662	0.735	126.12
0.20	1.786	0.721	147.71	1.477	0.634	132.97
0.25	1.338	0.642	108.41	1.144	0.579	97.58
0.30	1.284	0.592	116.89	1.104	0.544	102.94
0.35	1.175	0.555	111.72	1.025	0.503	103.78
0.40	1.091	0.531	105.46	0.965	0.492	96.14
0.45	1.000	0.515	94.17	0.907	0.485	87.01
0.50	0.961	0.504	90.67	0.874	0.480	82.08
0.55	0.929	0.495	87.68	0.852	0.475	79.37
0.60	0.915	0.488	87.50	0.813	0.472	72.74
0.65	0.856	0.483	77.23	0.803	0.469	71.21
0.70	0.825	0.479	72.23	0.783	0.467	67.67
0.75	0.815	0.475	71.56	0.776	0.465	66.68
0.80	0.804	0.472	70.34	0.769	0.463	66.09
0.85	0.794	0.469	69.29	0.762	0.462	64.94
0.90	0.774	0.468	65.38	0.750	0.461	62.29
0.95	0.766	0.466	64.38	0.745	0.461	61.60
1.00	0.761	0.459	65.79	0.741	0.457	62.14

Table 3.3 Effect of Different Velocity Spectra on Gust Ratios
(Example Building # 2)

Freq.	$\zeta = 0.010$		%	$\zeta = 0.016$		%
in Hz	Eq. 3.2.2.1	Eq. 3.2.2.3	Error	Eq. 3.2.2.1	Eq. 3.2.2.3	Error
0.05	1.820	1.000	82.0	1.450	0.823	76.18
0.10	1.280	0.689	85.78	1.040	0.585	77.78
0.15	0.987	0.549	79.78	0.815	0.483	68.74
0.20	0.802	0.457	75.49	0.677	0.432	56.71
0.25	0.692	0.419	65.16	0.596	0.403	47.89
0.30	0.600	0.397	51.11	0.530	0.386	37.31
0.35	0.568	0.384	47.92	0.500	0.375	33.33
0.40	0.507	0.375	35.20	0.470	0.369	27.37
0.45	0.480	0.370	29.73	0.440	0.364	20.88
0.50	0.456	0.365	24.93	0.436	0.361	20.78
0.55	0.452	0.362	24.86	0.433	0.359	20.61
0.60	0.452	0.360	23.55	0.430	0.357	20.45
0.65	0.440	0.357	23.25	0.424	0.355	19.44
0.70	0.433	0.356	21.63	0.416	0.354	17.51
0.75	0.374	0.355	5.35	0.361	0.353	2.27
0.80	0.366	0.354	3.39	0.356	0.352	1.14
0.85	0.363	0.353	2.83	0.354	0.352	0.57
0.90	0.361	0.352	2.56	0.352	0.352	0.0
0.95	0.357	0.351	1.71	0.350	0.351	-0.28
1.00	0.354	0.351	0.85	0.348	0.351	-0.85

Table 3.4 Effect of Different Velocity Spectra on Gust Ratios
(Example Building # 3)

Freq. in Hz	$\zeta = 0.010$ Eq. 3.2.2.1	$\zeta = 0.010$ Eq. 3.2.2.3	% Error	$\zeta = 0.016$ Eq. 3.2.2.1	$\zeta = 0.016$ Eq. 3.2.2.3	% Error
0.05	1.470	0.933	57.55	1.180	0.722	63.43
0.10	1.015	0.599	69.45	0.826	0.515	60.39
0.15	0.786	0.481	63.41	0.652	0.430	51.63
0.20	0.641	0.418	53.35	0.544	0.380	43.16
0.25	0.556	0.377	47.78	0.481	0.361	33.24
0.30	0.500	0.364	37.36	0.440	0.351	25.36
0.35	0.476	0.330	44.24	0.425	0.351	21.08
0.40	0.439	0.330	33.03	0.400	0.330	21.21
0.45	0.426	0.329	29.48	0.390	0.328	18.90
0.50	0.394	0.328	20.12	0.368	0.329	11.85
0.55	0.386	0.328	17.68	0.363	0.328	10.67
0.60	0.373	0.327	14.07	0.354	0.327	8.26
0.65	0.364	0.327	11.31	0.349	0.327	6.73
0.70	0.354	0.327	8.26	0.342	0.327	4.59
0.75	0.345	0.327	5.50	0.339	0.327	3.67
0.80	0.339	0.327	3.67	0.336	0.327	2.75
0.85	0.335	0.328	2.13	0.333	0.327	1.83
0.90	0.332	0.327	1.53	0.330	0.327	0.92
0.95	0.331	0.327	1.22	0.327	0.327	0.00
1.00	0.327	0.327	0.00	0.325	0.327	-0.61

Table 3.5 Effect of Different Velocity Spectra on Gust Ratios
(Example Building # 4)

Freq.	$\zeta = 0.010$		%	$\zeta = 0.016$		%
in Hz	Eq. 3.2.2.1	Eq. 3.2.2.3	Error	Eq. 3.2.2.1	Eq. 3.2.2.3	Error
0.05	0.846	0.500	69.20	0.682	0.427	59.72
0.10	0.519	0.330	57.27	0.432	0.309	39.81
0.15	0.398	0.264	50.75	0.298	0.265	12.45
0.20	0.336	0.264	27.27	0.271	0.265	2.27
0.25	0.296	0.263	12.55	0.257	0.262	-1.91
0.30	0.276	0.263	4.94	0.249	0.263	-4.96
0.35	0.264	0.262	0.76	0.240	0.263	-8.75
0.40	0.250	0.262	-4.58	0.235	0.261	-9.96
0.45	0.243	0.261	-6.89	0.232	0.261	-11.1
0.50	0.239	0.261	-8.43	0.230	0.261	-11.87
0.55	0.236	0.261	-9.57	0.227	0.261	-12.30
0.60	0.231	0.261	-11.49	0.226	0.261	-13.41
0.65	0.230	0.261	-11.87	0.228	0.261	-12.1
0.70	0.227	0.261	-12.3	0.225	0.261	-13.70
0.75	0.227	0.261	-12.30	0.224	0.261	-14.18
0.80	0.225	0.261	-13.7	0.223	0.261	-14.56
0.85	0.224	0.261	-14.18	0.223	0.261	-14.56
0.90	0.223	0.261	-14.56	0.222	0.261	-14.94
0.95	0.223	0.261	-14.56	0.222	0.261	-14.94
1.00	0.222	0.261	-14.94	0.222	0.261	-14.94

variation of gust ratios with the fundamental frequency of the building (see Figs. 3.3–3.6).

It is seen from the figures that the dependence of gust ratios in any example building on the natural frequency is almost identical in both the approaches. However, the gust ratios as estimated by using the height-independent spectrum are much higher than those based on the Eq. (3.2.2.3), especially for the long period buildings. As shown in Fig. 3.7, this is due to the fact that the height-independent spectrum overestimates the energy in velocity fluctuations in the relevant frequency range. For very stiff structures, the observed differences become much smaller. Further the gust ratios approach constant values beyond certain frequencies thus indicating the response to be pseudo-static. The frequencies beyond which the pseudo-static response is obtained, is governed by the band of energy distribution in the force spectrum, which shifts towards the lower frequencies for the buildings with larger dimensions, for a given velocity spectrum. Larger is the building, smaller will be the value of this frequency. This happens due to the decreased velocity correlation between any two points at higher frequencies. For any given building, for the same reason, convergence to the constant value is obtained at the higher values in the case of height-independent spectrum (see Fig. 3.3). From the above, it follows that the effects of gusts are likely to be important in most of the cases when the structure natural frequency does not exceed 0.6 Hz. In this range, the overconservatism resulting from the use of height-independent spectra may be as much as 60 percent for typical tall buildings with time periods close to 10 seconds (see Fig. 3.6). This, however, gets reduced for the taller buildings as can be seen from the comparison of gust ratios for different buildings with the same fundamental frequency. Further, as

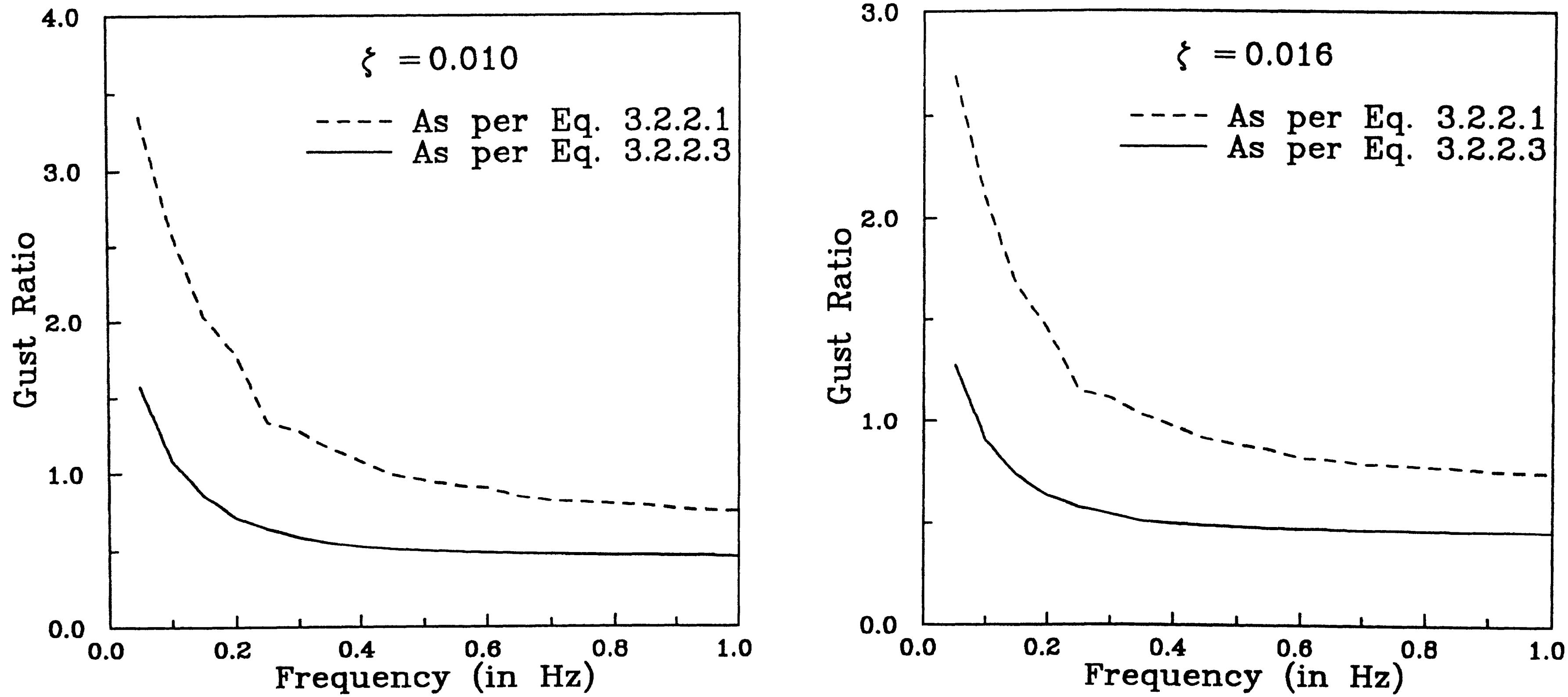

Fig. 3.3 Variation of Gust Ratios for Different Velocity Spectra (Building # 1).

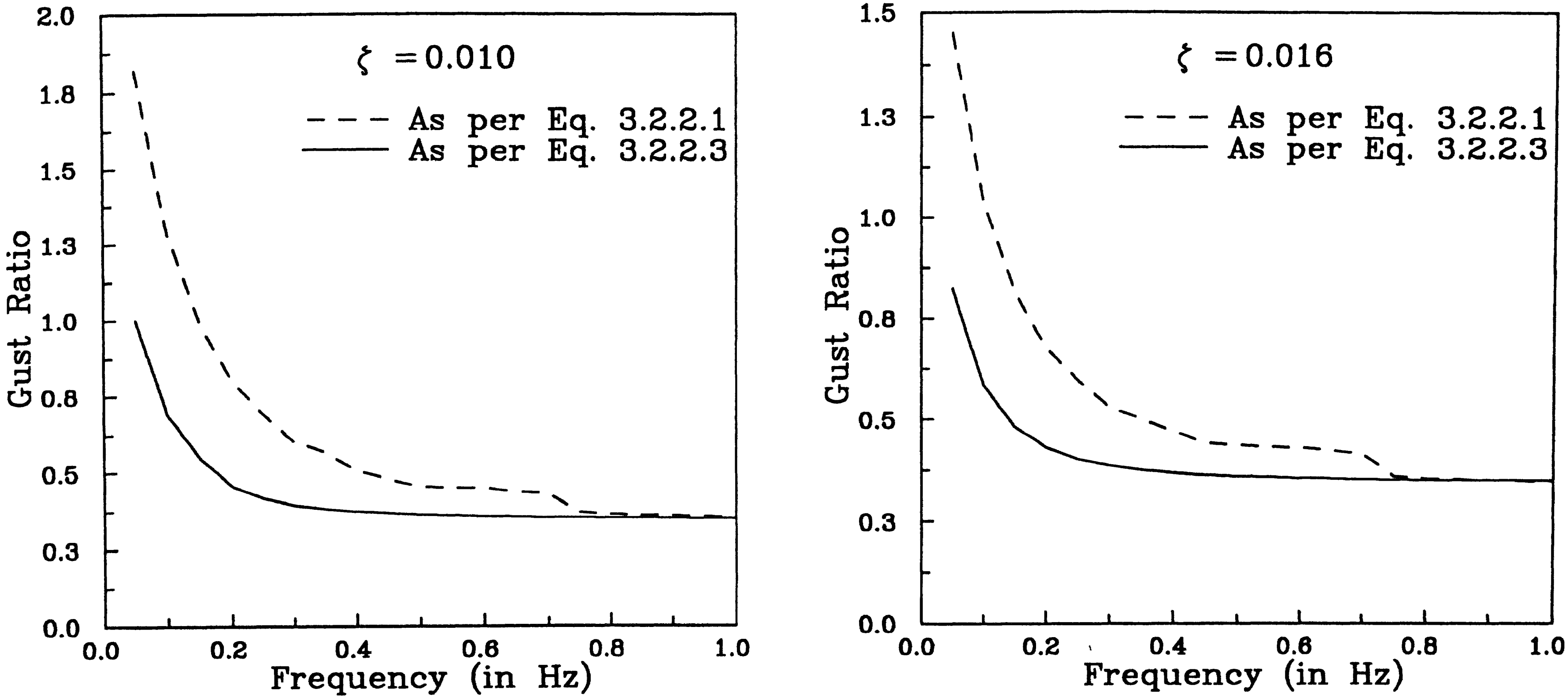

Fig. 3.4 Variation of Gust Ratios for Different Velocity Spectra (Building # 2).

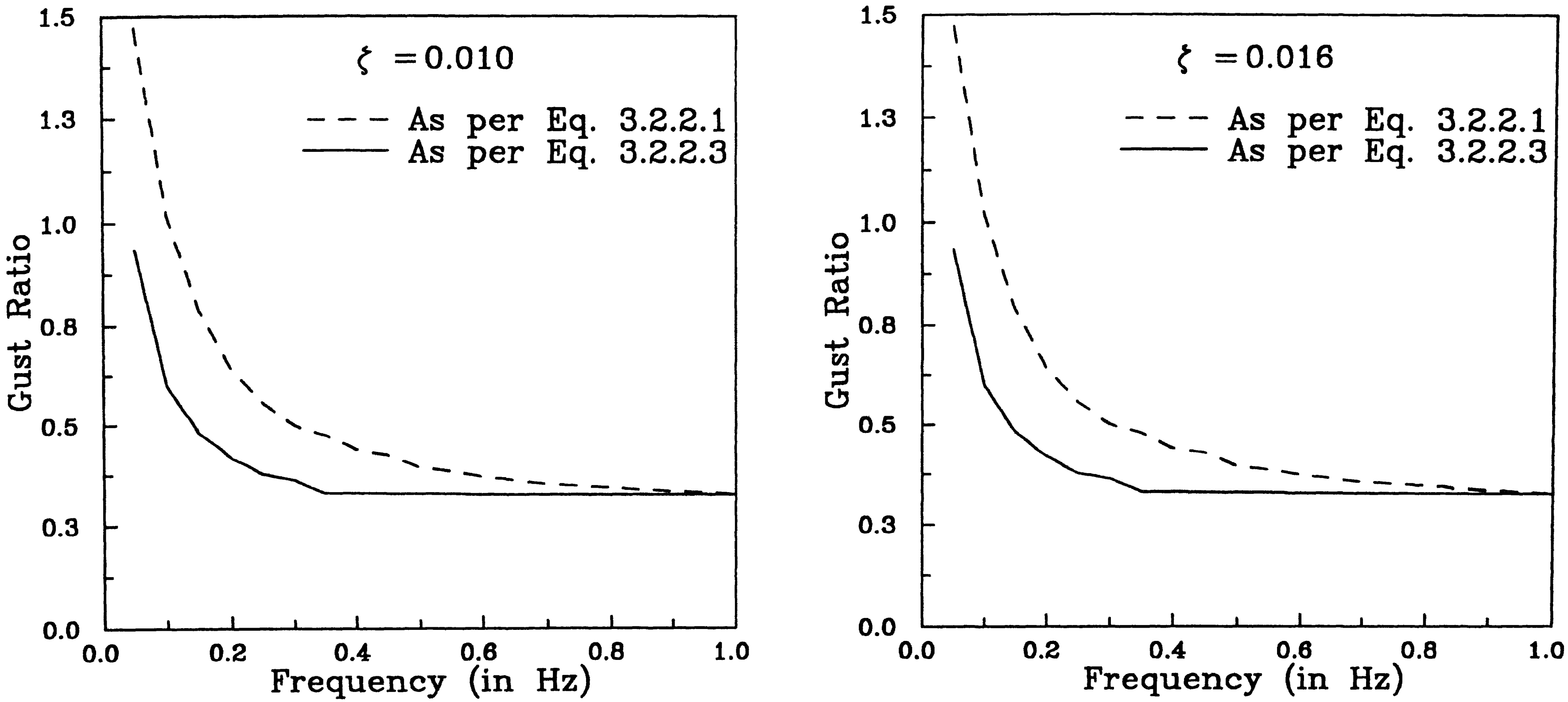

Fig. 3.5 Variation of Gust Ratios for Different Velocity Spectra (Building # 3).

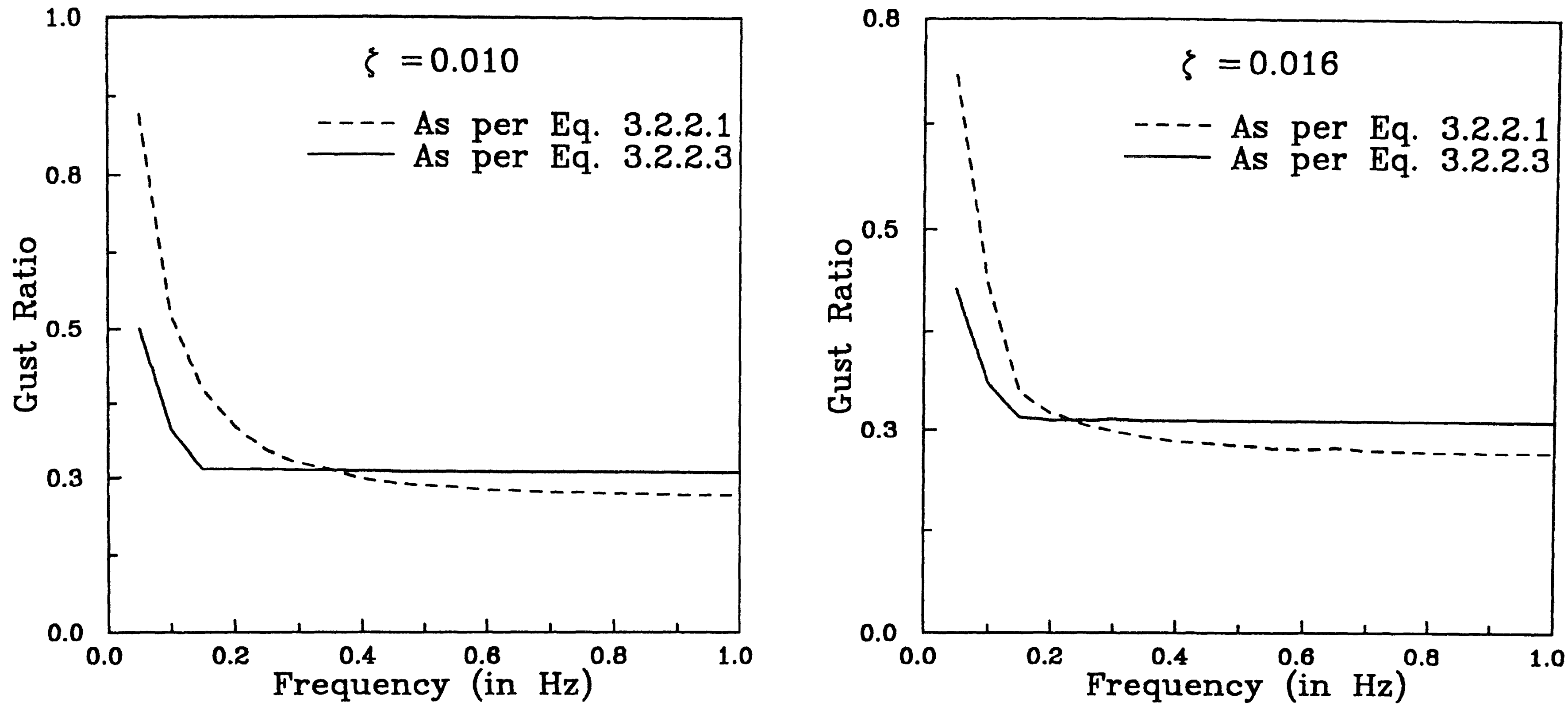

Fig. 3.6 Variation of Gust Ratios for Different Velocity Spectra (Building # 4).

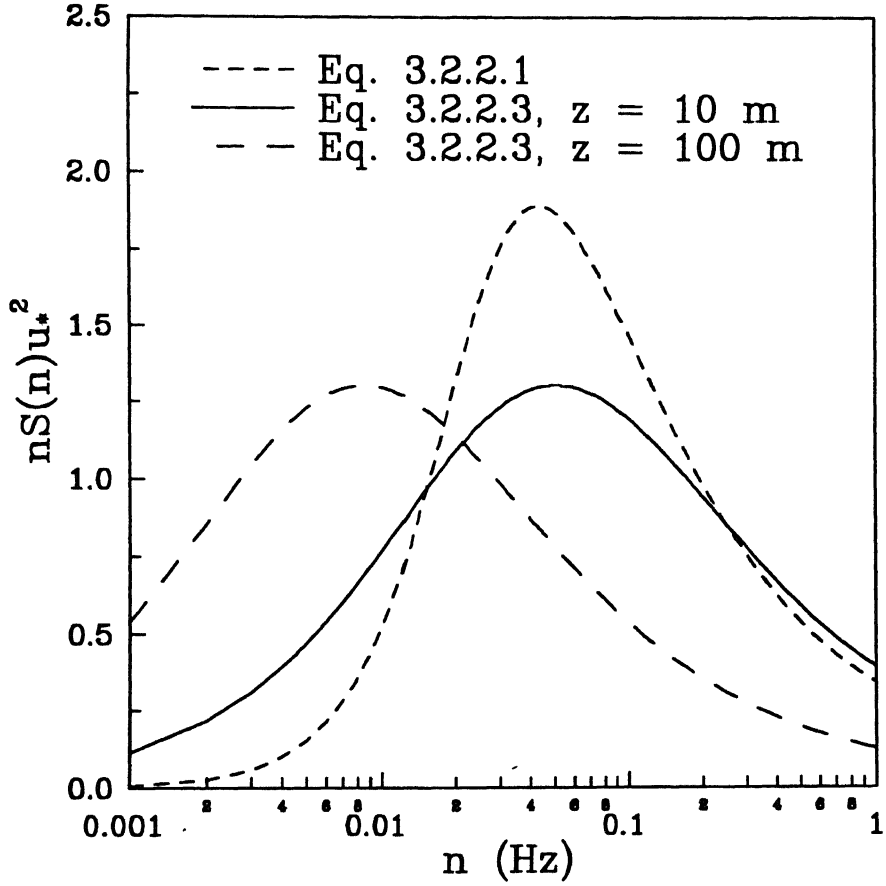

Fig. 3.7 Comparison of Different Velocity Spectra.

expected, all of these figures show reduced gust ratios for the higher damping ratios. The code provisions seem to properly account for these effects.

3.2.3 Mean Velocity Profile

As per the code, the mean wind velocity up to 10 m is assumed to be invariant with height. Above 10 m, the mean wind profile is modelled by the power law where, the mean wind velocity, $\bar{u}(z)$ at any height is given by

$$\bar{u}(z) = \bar{u}(10)\left[\frac{z}{10}\right]^{\alpha}. \qquad (3.2.3.1)$$

Here, $\bar{u}(10)$ is the mean wind velocity at height 10 m, z is the height in metres and α is the power law coefficient which depends on the terrain roughness. Recent research (Simiu (1976)) has, however, shown that the logarithmic law as expressed by

$$\bar{u}(z) = 2.5u_* \ln \frac{z - z_d}{z_o} \qquad (3.2.3.2)$$

is a more realistic representation of the mean wind profile. In this, z_d is called the zero plane displacement, z_o is the roughness length, and u_* is the friction velocity. The values of z_o and z_d depend on the terrain roughness. z_d represents the height above the ground level up to which there are no fluctuations in the wind flow due to the roughness elements, and thus it can be assumed zero everywhere except in the center of large cities where its value is generally taken as 20 m. Up to the height, z_d, the logarithmic law assumes the velocity to be constant. The use of logarithmic law for a mean wind profile has the following additional advantages. First, the logarithmic law is universally employed in the meteorological work, and therefore, the results of meteorological research can be readily employed in the engineering research. Errors inherent in reformulating such results in terms

of the power law are thus avoided. Secondly, in the logarithmic representation, the mean wind speed just as in the case of velocity spectra (Eq. 3.2.3.3) is an explicit function of u_*. This ensures an internally consistent representation of the flow structure.

Fig. 3.8 shows the variation in gust ratios in building #3 (with $\zeta = 0.01$) as obtained by taking the mean wind profile as logarithmic and that described by the power law. For the case of power law, two cases have been considered, one with the height-independent spectrum and another with the height-dependent spectrum. For the gust ratios based on logarithmic law, height-dependent spectrum has been considered. As the datum value, the mean wind speed at $z =$ 10 m, has been taken same in both the profiles. It is seen from the figure that the logarithmic law gives consistently smaller values of gust ratios. This can be understood further from Fig. 3.9 in which the logarithmic law gives lower wind speeds for all wind heights greater than 10 m. For the results in this study, however, the power law variation has been retained for consistency with the code provisions.

3.3 Approximations in Alongwind Response Calculations

Apart from the assumptions made in the Indian Standard code, there are certain fundamental assumptions which have been made in the formulation of gust factor (see Chapter II). In this section, the errors associated with some of these assumptions are estimated for some example cases to examine their validity.

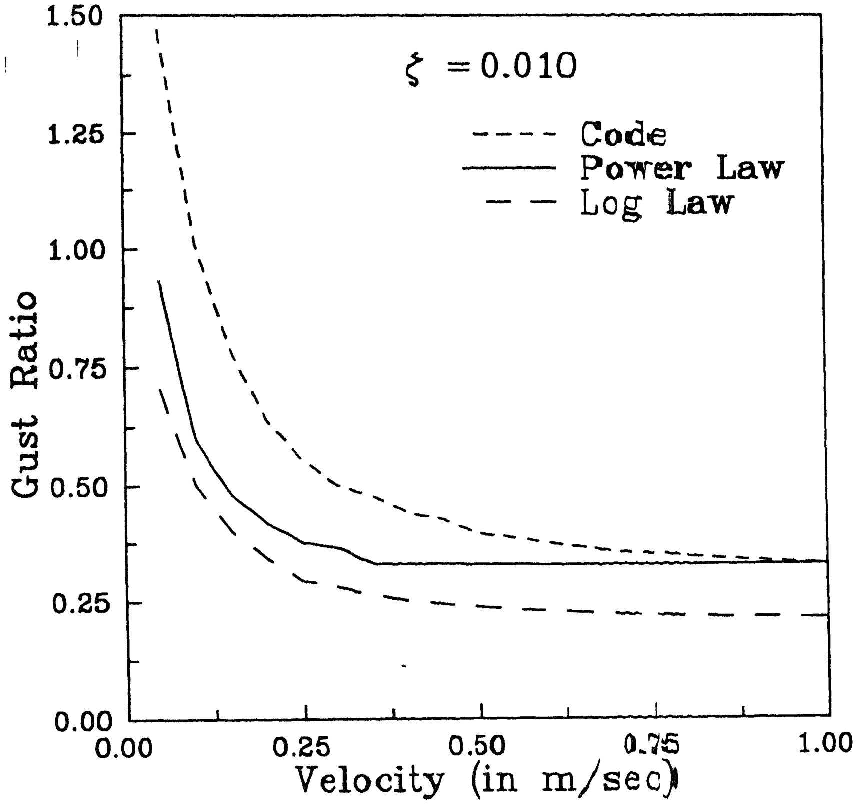

Fig. 3.8 Variation of Gust Ratios for Different Velocity Profiles.

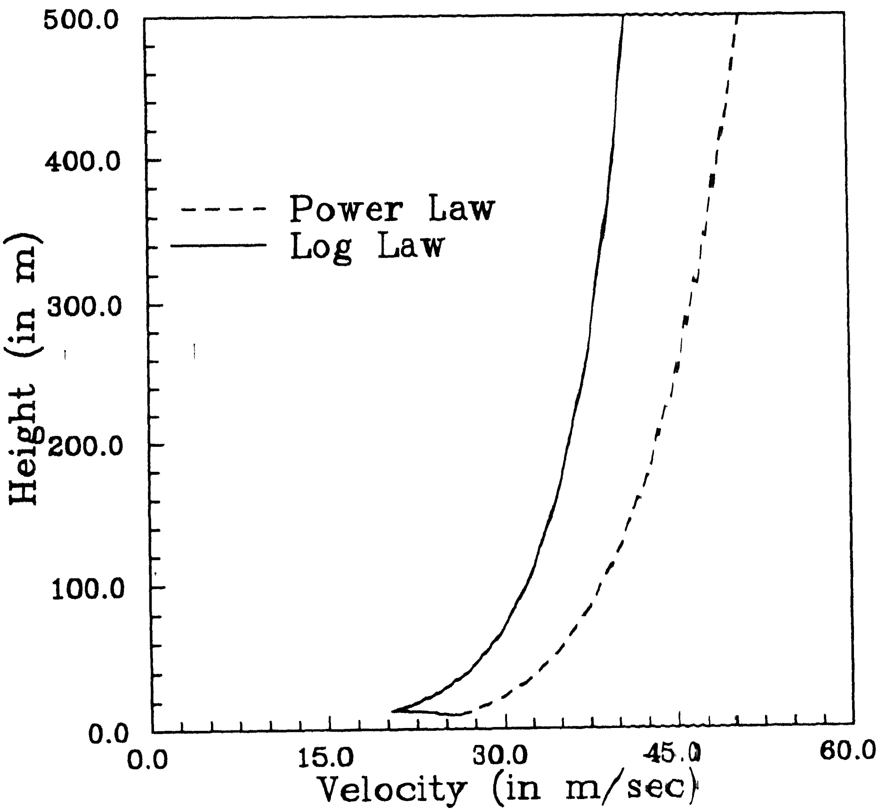

Fig. 3.9 Comparison of Mean Wind Velocity Profiles.

3.3.1 Linear Fundamental Mode Shape

Vickery (1970) has shown that for a mean wind profile described by the power law, the assumption of a linear mode shape is reasonably good. To investigate the change in gust ratio due to the deviation of mode shape from straight line, in a logarithmic profile, the mode shape $\mu(z)$ at any height, z has been described as

$$\mu(z) = \left(\frac{z}{H}\right)^{\gamma} \tag{3.3.1.1}$$

where γ is a parameter which can be varied. In the case of a linear mode shape, the value of γ is equal to be 1.0. On varying the value of γ between 0.5 and 2.0, in case of all example buildings, no appreciable change in value of gust ratio has been observed.

3.3.2 Contribution of Higher Modes

In the formulation of gust factors, it has been assumed that the fundamental mode primarily contributes to the building response. To investigate the error involved in this assumption, Simiu (1974) calculated the gust factors for the first and second modes assuming them to be linear and piecewise linear respectively. In this study, the gust ratios are calculated for the example buildings #3 and #4 by considering the first three modes only. The mode shapes are taken the same as in the case of shear buildings, and are thus described by the sinusoidal curves i.e. by $\mu(z) = \sin(2r-1)\pi z/H$ where, r represents the mode number and H, the height of the structure. Further, on assuming the first three natural frequencies in 1:3:5 ratio, it is found that the contribution of second and third modes to the gust ratios is less than 0.1%. The reasons for this can be

understood by examining the term for the fluctuating response i.e., $\sigma_y(z)/\bar{y}(z)$ in the expression for gust factor. The expression for $\sigma_y(z)$ in case of the response being described by r number of modes can be written as (see Appendix)

$$\sigma_y(z) = \left[\sum_r \int_0^\infty |H_r(n)|^2 S_{F_r}(n)\, dn\right]^{1/2}. \tag{3.6.2.1}$$

It is obvious that the value of $|H_r(n)|^2$ decreases rapidly for increasing natural frequencies with the number of the mode. In addition to this for the wind excited structures, the modal force $S_{F_r}(n)$ (see Eq. 2.5.11) is much higher for the fundamental mode shape compared to the higher mode shapes. This is due to the resemblance between the first mode shape and the distribution of mean wind pressures acting along the height which does not lead to any mutual cancellation of the wind pressure contributions at different heights to the total energy in the mode. In higher mode shape, however, such cancellation causes modal energy to be reduced to a very small value. These observations are parallel to those of Gupta and Trifunac (1989) in their study of base rocking due to the soil-structure interaction in seismic reponse of multistorey buildings. Thus, the total contribution of higher modes to $\sigma_y(z)$ is likely to be very less even when the modal frequencies are very closely spaced. The same reasons hold good for the dominance of first mode in $\bar{y}(z)$ where the expression for $\bar{y}(z)$ for r modes can be written as

$$\bar{y}(z) = \sum_r \frac{\int_{A_w} \bar{p}_w(x,z)\mu(z)\, dA + \int_{A_l} \bar{p}_l(x,z)\mu(z)\, dA}{(2\pi n_r)^2 M_r}. \tag{3.6.2.2}$$

CHAPTER IV
CONCLUSIONS

The dynamic alongwind response of buildings using the gust factor method has been studied here. A review of this method as adopted by the Indian Standard code (IS: 875-1987) has shown that the recommended gust factors are too conservative due to the use of incorrect velocity spectra and alongwind correlation factor. It has been observed that the gust factors approach constant values for the stiffer buildings, thus indicating the response to be pseudo-static. The frequency beyond which the pseudo-static response is obtained for a building, is governed by the band of energy distribution in the force spectrum. Larger the building, smaller is the value of this frequency. It has also been seen that the logarithmic mean wind profile leads to lower gust factors in comparison with those specified by the code. Further, the assumption of a linear mode shape for the calculations of gust factors, in case of the mean wind velocity profile being described by a logarithmic law, has been justified. It has also been shown that the contribution of higher modes to the dynamic alongwind response is negligible in comparison with that of the fundamental mode.

This study has been conducted on the assumption that the response of structures due to wind is basically in the alongwind direction. For very tall structures, however, the acrosswind response may dominate the alongwind response. In addition, torsional response may also be quite appreciable for certain building configurations. These effects need to be included in a more comprehensive and rational study of the wind response of the buildings.

APPENDIX

In this appendix, the spectral density for the displacement response of a continuous system has been derived.

Consider a continuous structure as shown in Fig. 2.2. For this, it may be assumed that the displacement in the y-direction is the same for all points having the same ordinate, z. Further, assuming that the mass, damping and stiffness of the structure are uniform throughout, the equation of motion can be written as

$$m\frac{\partial^2 y}{\partial t^2} + c\frac{\partial y}{\partial t} + k\frac{\partial^4 y}{\partial z^4} = P(x, y, z, t), \tag{1}$$

where, $P(x, y, z, t)$ represents a random fluctuating force with spectral density $S_P(x, y, z; n)$. On using the normal mode approach in Eq. (1) and determining the transfer function, the spectral density of displacement, $S_y(z, n)$ can be written as

$$S_y(z, n) = \sum_{j=1}^{\infty}\sum_{k=1}^{\infty} \frac{\mu_j(z)\mu_k(z)}{M_j M_k} H_j(n) H_k^*(n) S_{Pjk}(n), \tag{2}$$

where $\mu_j(.)$ represents the mode shape. $H_j(n)$ is the complex frequency response function, and M_j is the modal mass in the j^{th} mode. $H_k^*(n)$ is the complex conjugate of $H_k(n)$. $H_j(n)$ is defined by the expression

$$H_j(n) = \frac{1}{4\pi^2\left[n_j^2 - n^2 + 2i\zeta_j n n_j\right]}, \tag{3}$$

where n_j and ζ_j are the natural frequency and damping of the j^{th} mode. The term $S_{Pjk}(n)$ in Eq. (2) is called the cross modal force as defined by

$$S_{Pjk}(n) = \int_A \int_A \mu_j(z_1)\mu_k(z_2) S_P(P_1, P_2; n)\, dA_1\, dA_2 \tag{4}$$

where, A is the total area over which the force $P(x, y, z, t)$ is supposed to act and $S_P(P_1, P_2; n)$ is the cross-spectral density of this force between two points

P_1 and P_2 of co-ordinates (x_1, y_1, z_1) and (x_2, y_2, z_2) and having elemental areas dA_1 and dA_2 respectively. In case of the wind loads on buildings, the forces on side faces are ignored. Further, it is assumed that the velocity fluctuations on the windward and leeward faces are similar. Hence, the force $P(x, y, z, t)$ in Eq. (1) becomes independent of y. Its value at any point is given by

$$P(x, z, t) = \rho c_p(x, z)\bar{u}(z)u_t(x, z), \tag{5}$$

where $c_p(x, z)$ represents the pressure coefficient, and $\bar{u}(z)$, $u_t(x, z)$ are the mean and fluctuating velocities at a point with co-ordinates (x, z). Thus, the cross-spectral density of $P(x, z, t)$ for any two points with co-ordinates (x_1, z_1) and (x_2, z_2) becomes

$$S_P(x_1, x_2, z_1, z_2; n) = \rho^2 c_p(x_1, z_1)c_p(x_2, z_2)\bar{u}(z_1)\bar{u}(z_2)S_u(x_1, x_2, z_1, z_2; n). \tag{6}$$

The term $S_u(x_1, x_2, z_1, z_2; n)$ here represents the cross-spectral density of velocity fluctuations, $u_t(x, z)$. Its value as given by Davenport (1967) is

$$S_u(x_1, x_2, z_1, z_2; n) = S_u^{1/2}(z_1, n)S_u^{1/2}(z_2, n)R(x_1, x_2, z_1, z_2; n)N(n), \tag{7}$$

where $S_u(z, n)$ is the spectrum of velocity fluctuations, and $R(x_1, x_2, z_1, z_2; n)$ and $N(n)$ are called the acrosswind and alongwind correlation factors.

If the damping of the structure is small and the modes are widely spaced apart, the cross-modal force S_{Pjk} can be neglected for $j \neq k$. Further, assuming that the fundamental mode contributes primarily to the response, Eq. (2) reduces to

$$S_y(z, n) = \frac{\mu^2(z)|H(n)|^2}{M^2} \int_A \int_A \mu(z_1)\mu(z_2)S_P(P_1, P_2; n)\, dA_1\, dA_2. \tag{8}$$

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